A MODEL STUDY OF THE HYDRAULIC STRUCTURES AT THE CONFLUENCE OF THE SAN TOMAS AQUINO AND SARATOGA CREEKS

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(1)

Mr Fowler:

(1.) The model will be available next semester.

(2.) The weir on the top of the drop is necessary to

a. stabilize the jump in the basin by making it one of greater momentum differential. (We did remove this weir and the model did not function as well as with it). Professor Murray refers to this weir on page 10 third paragraph. and on page 13 the weir is recommendation no. 1.

(3.) It would be simple to run the test suggested with the corner removed but I don't think that I would suggest holding up construction for this test. The waves produced are the result of the higher momentum (higher flows) in Saratoga Creek. I don't see any economic advantage in this modification since the wall heights have to be raised 1½ feet (Recommendation 5, page 14)

(4) As a matter of discussion I don't agree with the ~~hypothesis~~ author's theory of wave generation downstream from transitions. Waves generated at an outlet transition can only result from transition flow that is unstable (Flow depth near critical depth). This is not the case in the San Tomas transition. This, of course, does not apply to small boundary waves that are not significant. ~~This~~ We have always attempted to design our culvert transitions (outlet) so that a stable depth of flow results in the subcritical range.

I am in favor of offset printing the report providing proper credits are given to the people who designed the structure and guided it through Professor Murray and the tests.

(a) I am sure the University will agree but I will get a letter of approval.

(b) I recommend we redo the drawings and improve them for printing. The pictures are in our files.

(c) Definitely, the profiles must be reworked but I think this could be done by our people along with the other drawings.

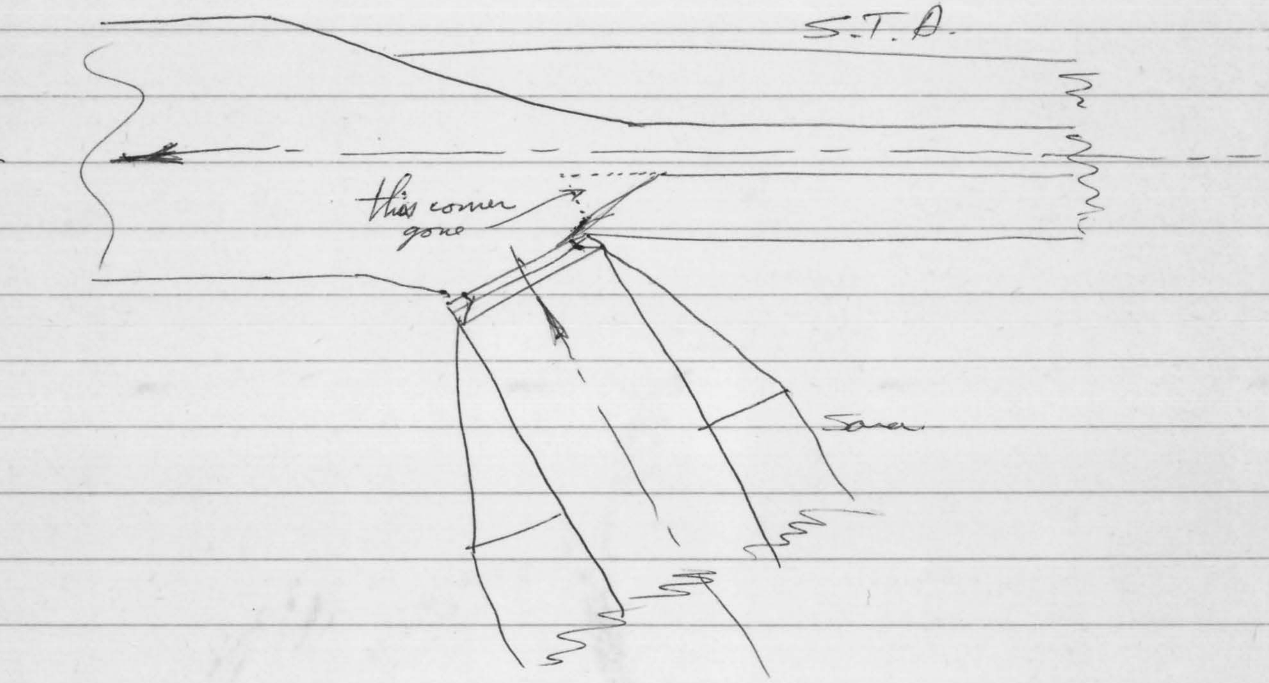
Charlie Battersby

It is a good report -

Is the model still available?

a) If so would you check the need for the flip weir on the top of the drop structure.

b) How about changing the S.T.A. inlet to the upper basin so as to have a similar upstream vertical face as the opposite side has. This might help the east bank swirl?



Are the negatives of the pictures available?

I think we should reproduce this report by offset

a) Would this be OK by University?

b) Are originals of drawings available?

c) The profiles need some better definition for reproduction

Charles -

Good points - Will you please arrange for modifications in the report you suggest (see if they are OK with S.C.U.) and we will reproduce report.

L.

Al Bacher
484-8202
Bridge Dept.
Special Design.

A MODEL STUDY OF THE HYDRAULIC STRUCTURES AT THE CONFLUENCE OF THE SAN TOMAS AQUINO AND SARATOGA CREEKS

Prepared for the Santa Clara County
Flood Control and Water District

by

R. Ian Murray
Assistant Professor of Engineering
University of Santa Clara
June 1965

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ABSTRACT

A model study of the proposed channel improvements for flood control in the San Tomas Aquino and Saratoga Creeks was conducted to check the proposed hydraulic design and to determine any needed modifications where unsatisfactory conditions were disclosed by the model.

The proposed design incorporated a drop structure below the confluence basin and a stilling basin below the drop structure. It was found that energy dissipation in the stilling basin was inadequate at 2300 and 2500 cfs (25-28% of design flow) and that the stability of the hydraulic jump was poor when the tailwater was lowered at the design flow rate of 6400 cfs.

These adverse conditions were significantly improved by placing a weir on top of the drop structure and an end sill about 100 feet downstream from it.

It was also found that waves were generated by the transition section at the end of the stilling basin. Since this section has already been constructed, the most feasible action is to employ riprap to protect the sides of the unlined channel.

ACKNOWLEDGMENT

The author gratefully acknowledges the assistance provided by Robert A. George, Project Engineer, and by other members of the staff of the Santa Clara County Flood Control and Water District. He also wishes to thank David Robertson, senior civil engineering student, for designing the model and executing the test program, Frank Piggott for constructing the model, and Michael Boudreau and Gloria Usher for their assistance in preparing this report.

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INTRODUCTION

The project for which this model study was undertaken is the channel improvement at the confluence of the San Tomas Aquino Creek and the Saratoga Creek. The confluence occurs approximately 270 feet upstream from the Monroe Street bridge that crosses San Tomas Creek. The channel is already lined in the vicinity of the bridge; so the project extends from STA. 183 + 04.62 to STA. 185 + 52.33 of San Tomas Aquino Unit V. It also includes the transition section at the outfall of Saratoga Creek.

The proposed design is shown in Figs. 1 and 2. Below STA. 181 + 35.02 the channel is unlined; above STA. 185 + 52.33 the channel is to be lined as part of another project. Saratoga Creek is unlined. Existing lining and lining to be placed is portland-cement concrete.

The project is located in a residential area; thus the protection of life and property is of paramount importance. Because of the uncertainty in the available analytical methods for the hydraulic design of a confluence with supercritical flow and a large angle of convergence, a model study of the proposed design was deemed advisable.

The following objectives were established:

1) Verify the adequacy of the proposed design for all reasonable combinations of flow in the two channels.
2) Propose and test modifications of the design to correct any defects that might be discovered.

3) Propose and test modifications which might reduce the cost of construction.

4) Obtain water surface profiles for the final design.

The design flow rate for the project is 6400 cfs (2800 in San Tomas Creek and 3600 in Saratoga Creek). This flow was the predicted value for a 100 year flood. A revised prediction indicated the possibility of 7500 cfs (3500 in San Tomas Creek and 4000 in Saratoga Creek). The 6400 cfs value was kept as the design flow, but the model was tested up to 7500 cfs.

Analysis of the design indicated that a uniform flow could be anticipated in the Saratoga Creek, and an M2 type surface profile* could be anticipated upstream from the confluence in the San Tomas Creek. A control (critical depth) occurs at the outfall of Saratoga Creek, and a control should exist near STA. 185 + 53 on San Tomas Creek if it isn't drowned out by the flow from Saratoga Creek. From the confluence to the drop structure at STA. 184 + 41.38 the flow will be supercritical, and a hydraulic jump should form below the drop structure, followed by an M1 type surface profile to join the uniform flow in the unlined channel below STA. 181 + 35. The model was designed and tested on the assumption that this general description of the flow would be realized.

The critical item in the above description is the existance of uniform flow in the lower reach of San Tomas Creek, for this determines the tailwater depth. A weir at STA. 170 + 35 insures the existance of this condition for the design flow.

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*V. T. Chow, "Open-Channel Hydraulics", McGraw-Hill, 1959.

THE MODEL

Essentially this is a study of the performance of a stilling basin. Experience has shown that such studies can be performed in accordance with the Froude Law and with undistorted models, provided the scale is large enough to eliminate such extraneous effects as viscous and surface tension forces.* A 1:30 scale was adopted, for it fulfills this requirement and is compatible with the laboratory capabilities. The corresponding depth, velocity, and flow rate ratios are obtained by equating the Froude numbers of the prototype and model.

$$y_p/y_m = 30$$

$$V_p/V_m = 30^{\frac{1}{2}} = 5.48$$

$$Q_p/Q_m = 30^{5/2} = 4950$$

Complete similarity requires that the surface roughness as well as the overall geometry of the structure be modeled. This is obviously impossible; but an equivalent effect can be achieved if the Manning n is modeled, based on its dimension of length to the one-sixth power. The roughness coefficient ratio should then be

$$n_p/n_m = 30^{1/6} = 1.764$$

The value of n used in design calculations for the lined channels is 0.015. The corresponding value for the model would be 0.0085. This would require that the model be built of lucite which was not economically feasible for this study. An alternative would be to distort the slope of the model; but the amount of distortion would depend on the value of n actually achieved in the model, and this could only be determined by extensive verification tests with a variable slope model. This was not economically feasible either.

---

*"Hydraulic Models", American Society of Civil Engineers, 1942.

Rather than guess at the proper distortion, it was decided that an undistorted model should be built and made as smooth as possible. The fact that the roughness coefficient is not modeled can be taken into account in interpreting the results. This decision is further supported by the fact that changes of specific energy produced by the drop structures, the hydraulic jump, and the turbulence in the confluence basin are large compared with the changes produced by surface roughness. The influence of surface roughness should therefore be negligible in this study.[*]

With controls normally existing at the entrances to the confluence basin, the only requirements for the flows entering the model were that they be subcritical and of controllable magnitude. This was achieved with short reaches of channel upstream from the confluence, forebay tanks, and well flared entrance sections. The entrance section for the San Tomas Creek is shown in Fig. 6. The entrance for the Saratoga Creek is similar. The forebay tanks were supplied from 4 inch pipes connected to the laboratory's constant head tank. Gate valves provided flow control, and rock baffles in the forebay tanks were very effective stilling devices.

---

[*] The value of n for the unlined channel downstream from the project is vitally important in determining the tailwater depth, but this reach of channel is not included in the model.

At the downstream end of the model a short reach of trapezoidal section was required in which to establish and measure normal depth for the unlined reach of channel. The depth was controlled by a series of adjustable screens transverse to the channel. The depth was measured by a point gauge at STA. 180 + 99. The installation can be seen in Fig. 11.

The model extended from STA. 180 + 15 to STA. 186 + 42 and included 144.5 feet of the Saratoga Creek measured from its outfall. It was constructed with a 5/8 inch plywood base set on 2 x 2 and 2 x 4 cross members spaced not more than 16 inches apart. These were supported by a heavy angle iron framework and shimmed to give the proper elevations and slopes. The angle irons were excessively heavy but they were available from surplus stock and had the advantage of insuring that slopes would not change when the model was loaded with water.

The sides of the model were built with 3/8 inch plywood supported on a system of knees and 1 x 2 stringers. It would have been better if 5/8 inch plywood had been used without stringers, for warping of the stringers created some construction problems. The transition sections were formed from horizontally laminated 3/4 inch sugar pine. Various stages of construction are shown in Figs. 3-6.

The model was coated with fiberglas resin to provide a smooth sealed surface. It was then painted with a gray epoxy paint to give good photographic characteristics. Extra freeboard was provided on the model; so a dark band was painted along the top of the walls to indicate the wall height of the prototype.

It will be noted that the model was built as a mirror image of the prototype. This was done to make the best use of available floor space in the laboratory.

INSTRUMENTATION AND TEST PROCEDURE

The measurements required in this study were measurements of flow rate and depth. A sharp crested rectangular weir was installed downstream from the model in the return channel of the laboratory. Its crest was 12 inches above the bottom of the channel and it was 23-9/16 inches broad. It is shown in Fig. 7 together with the 3/4 inch pipe that provided ventilation. A rock baffle upstream from the weir provided stilling action, and the head on the weir was measured with a point gauge 33 inches upstream from the weir. Maximum and minimum heads were 0.353 feet and 0.088 feet respectively.

Flow rates were computed from head measurements using Rehbock's formula for the weir coefficient. At approximately 0.5, 1.0 and 1.5 cfs the flow was checked by taking velocity grids with a current meter. These measurements indicated that the Rehbock coefficient was low by 7.1% at 1.5 cfs, low by 0.9% at 1.0 cfs, and high by 3.2% at 0.5 cfs. Since the reliability of this calibration was questionable (the velocities were not steady), and since the time schedule did not allow for more extensive calibrations, it was decided that the Rehbock coefficient should be used for the study. It should be noted that this is a conservative decision, for at high flow rates the actual flow in the model will be greater than that reported if the trend of the current meter calibration is correct.

With controls at the entrances to the confluence basin, a unique relationship exists between the flow rate and water surface elevation in the forebay tank of each creek. A hook gauge was installed in each forebay tank, and this relationship was determined experimentally for each creek. In operating the model the flow rate in each creek was set by adjusting the gate valves in the input pipes until the proper water surface elevations were achieved in the forebay tanks. The sum of the two flows was checked with the weir. It was found that whenever the proportion of the design flow in the Saratoga Creek exceeded the proportion of design flow in the San Tomas Creek, the control in the San Tomas Creek was drowned out. Under these conditions the flow in the San Tomas Creek was always set first.

For each flow rate tested the normal depth in the unlined reach of San Tomas Creek was computed from Manning's equation using the design value of $\underline{n}$, 0.030. By adjusting the screens at the end of the model, the equivalent depth was established at the gauging station. The flow characteristics in the model were then noted and photographs were taken.

Because of the inevitable uncertainty in the value of $\underline{n}$, the tailwater depth was gradually lowered for a series of representative flow rates until a condition of "washout" was judged to exist. Such a condition is shown in Fig. 8. The depth of the flow at the gauging station was then recorded. The reduction of tailwater depth before washout provides a measure of the stability of the hydraulic jump and of the tolerance that can be allowed in the value of $\underline{n}$.

RESULTS OF TESTS ON THE ORIGINAL DESIGN

The original design was tested at the flow rates given in the following table.

San Tomas C cfs	Saratoga D cfs	Combined Flow G cfs	Condition
3500	4000	7500	100 yr. flood
2800	3600	6400	Design flow
"	2700	5500	1:3/4 of des. flow
"	1800	4600	1:1/2
2100	3600	5700	3/4:1
"	2700	4800	3/4:3/4
"	1800	3900	3/4:1/2
"	900	3000	3/4:1/4
1400	3600	5000	1/2:1
"	2700	4100	1/2:3/4
"	1800	3200	1/2:1/2
"	900	2300	1/2:1/4
700	2700	3400	1/4:3/4
"	1800	2500	1/4:1/2
"	900	1600	1/4:1/4
"	0	700	1/4:0
0	900	900	0:1/4

For most flow conditions the design appeared satisfactory, but in a few cases some remedial action was indicated. The most unsatisfactory condition existed at flow rates of 2300 and 2500 cfs. The hydraulic jump formed properly in only half of the channel. On the west side the jump was undular, and the waves thus generated would travel far downstream. This condition is shown in Fig. 9.

At 7500 cfs the downstream water surface was rough and waves overtopped the sides of both the rectangular and transition sections of channel (Fig. 11). The stability of the jump was quite poor, for the tailwater depth could be lowered by only 7.9% before "washout" would occur. This corresponded to an n value of 0.026.

For all flow rates waves were generated at the downstream corners of the transition to the unlined channel. The effect was especially pronounced in the higher range of flows. Fig. 12 shows such a wave being formed at the design flow of 6400 cfs. Occasionally these waves overtopped the sides of the transition section at this flow rate.

The cause of this wave generation is undoubtedly a transition section that is too short. Nothing can be done to eliminate this situation, for the transition section has already been constructed; but fortunately the ground level at this section is above the walls of the transition section. Riprap should be used to protect the earthen channel.

DESIGN MODIFICATIONS AND TEST RESULTS

To improve the hydraulic jump in the 2500 cfs flow rate range, baffle piers were installed downstream from the drop structure. These were tried in various sizes and arrangements, but no combination was very satisfactory. Piers placed on the west side of the channel slightly upstream from the jump did create a normal, rather than an undular, jump; but at flow rates other than 2500 cfs a rough downstream water surface was produced. This type of modification was therefore rejected.

A weir to be placed on top of the drop structure in the San Tomas Creek was designed by R. A. George, Project Engineer. Its cross-section is shown in Fig. 16. Flow at 2500 cfs with this weir installed is shown in Fig. 13.

At all flow rates tested its performance was very satisfactory, except that the stability against "washout" was poor in the low flow rate range. Specifically, at 1600 cfs the tailwater could only be lowered 10.7%. This corresponds to an n value of 0.025.

The weir, of course, produced subcritical flow in the stilling basin, but the control at the Saratoga Creek outfall was never drowned out and the M2 type of surface profile in the San Tomas Creek never shifted to an M1 curve. Normal flow condtions upstream from the confluence were therefore not disturbed.

To improve stability, an end sill for the stilling basin was installed. Its cross-section is shown in Fig. 17. It was located 104 feet downstream from the drop structure, but this dimension is not critical. Performance with this end sill was excellent. A stable jump formed at the base of the drop structure, and a weak jump formed at the base of the sill. The downstream water surface was very smooth except at high flow rates, and even then it was quite acceptable. Stability was excellent as will be described subsequently. Within the stilling basin, i.e., between the drop structure and the end sill, waves overtopped the walls at 6400 and 7500 cfs. The end sill was also tested without the weir on the drop structure. Its performance was satisfactory, but not as good as with the weir.

The final modification to the original design was prompted by economic considerations. The warped transition section at the entrance of the San Tomas Creek to the confluence basin appeared to have little value. It was therefore replaced by plane surfaces as shown in Fig. 18. Fig. 14 is a photograph of the final design.

The hydraulic effect of this modification was to direct more of the flow to the east side of the channel and to set up a swirl in the confluence basin. The increased flow on the east side of the channel caused the mean water surface on the east side of the stilling basin to overtop the walls by 0.6 feet at 7500 cfs. At the design flow of 6400 cfs waves overtopped the walls of the stilling basin, but they did this with the warped transition too.

The swirl in the confluence basin produced a backflow on the west side. Silt and gravel will no doubt be deposited, thus creating a maintenance problem. This condition may have existed with the warped transition, but it was not specifically noted. If it existed, it was much less pronounced.

At low flow rates the confluence of the two streams was quite smooth with a small hydraulic jump at the base of the Saratoga Creek outfall. With increasing flow rates a complex pattern of standing waves developed and the flow welled up on the east wall of the confluence basin. At 7500 cfs only 0.6 feet of freeboard remained. A very strong eddy existed at the lower end of the hydraulic jump at the base of the Saratoga Creek outfall. There was no effect on the performance of the stilling basin except as already noted; there was no effect whatsoever downstream from the stilling basin. The flow pattern in the confluence basin at 6400 cfs can be seen in Fig. 20.

FINAL TESTS

Formal tests of the final design were made for balanced flow rates only. However, in the process of setting up these flows unbalanced flows were observed transiently and no unsatisfactory conditions were noted. Furthermore, experience had shown that unbalanced flows made no difference in the performance of the stilling basin; only the magnitude of the combined flow was significant.

The results of washout tests are given in the following table:

Flow	Tailwater Reduction	Manning's n
cfs	%	
7500	20.2	0.0197
6400	21.8	
4800	23.8	
3200	21.0	
1600	19.4	0.0206

In these tests, "washout" consisted of the weak jump at the base of the end sill moving downstream and becoming undular. The primary jump in the stilling basin remained stable!

This high degree of stability suggested that the height of the end sill might be reduced to eliminate the need for increasing the height of the side walls of the stilling basin. It was felt that to be effective in achieving this objective the height of the end sill would have to be reduced by a factor of two. When this was tried, the results were very unsatisfactory. The primary jump would not form naturally in the stilling basin, and the end sill acted as a flip bucket producing severe wave action downstream. It can be concluded that the height of the end sill should not be reduced without additional model tests to determine the limit of stability.

The excellent improvement in the performance of the stilling basin for the 2300 to 2500 cfs flow range can be thoroughly appreciated by comparing Figs 9 and 10. Photographs of the water surface profiles for 2500 cfs and 6400 cfs are shown in Figs. 20-24. A plot of water surface profiles measured along the center lines of the channels is given in Fig. 26. Fig. 25 shows that the generation of waves at the downstream corners of the transition to the unlined channel persists in the final design.

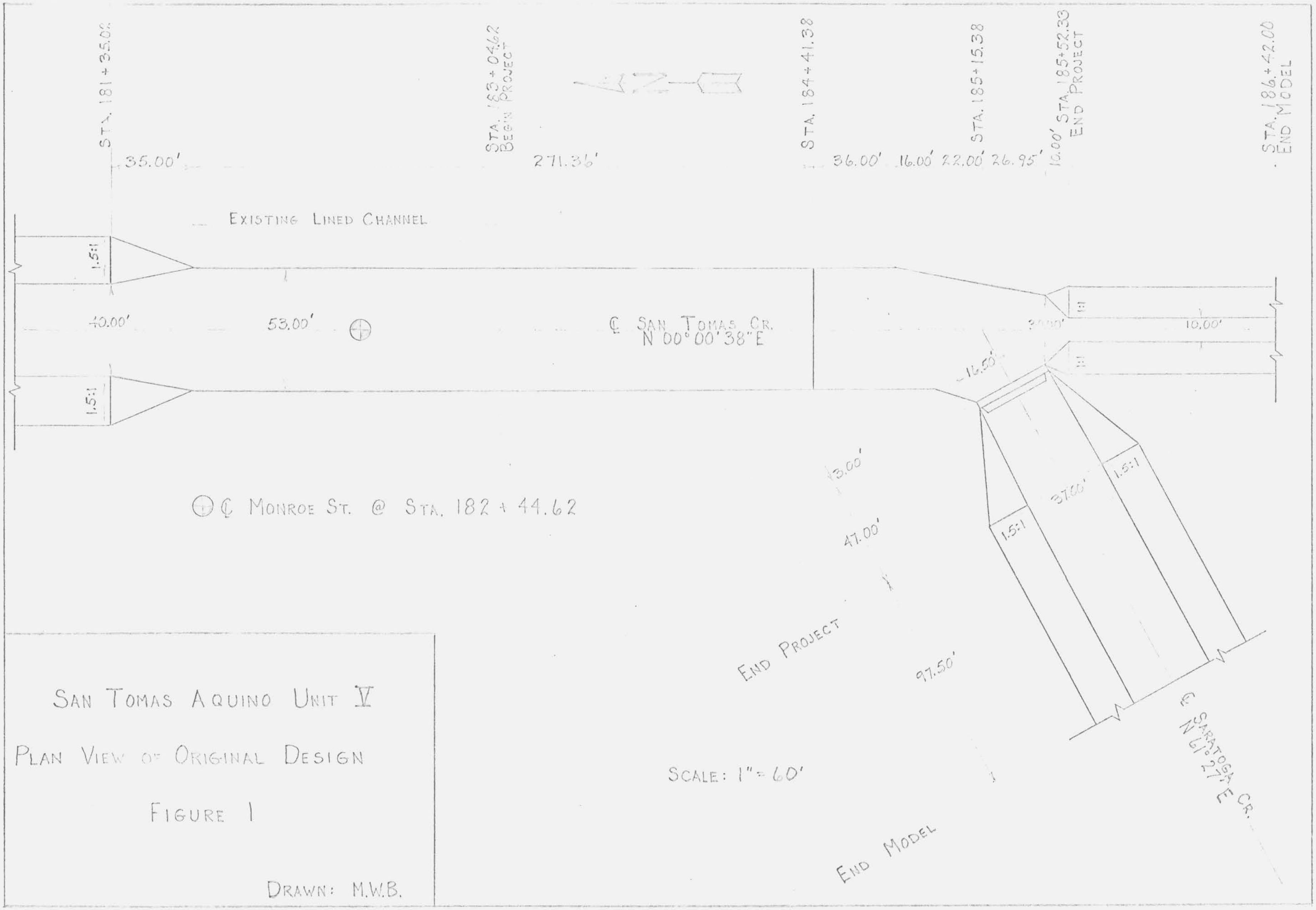
RECOMMENDATIONS

Based on the tests performed in this study, the following recommendations are made:

1. A weir, as proposed in the original design, should be constructed on top of the drop structure at the outfall of the Saratoga Creek. Its cross-section is shown in Fig. 15.
2. A weir having the cross-section shown in Fig. 16 should be constructed on top of the drop structure in the San Tomas Creek.
3. An end sill having the cross-section shown in Fig. 17 should be constructed approximately 100 feet downstream from the drop structure in the San Tomas Creek. Its crest could undoubtedly be flattened in conformance with normal construction practices without adverse effects. Its height should not be reduced without first performing additional model studies.

4. The decision whether to use a plane or warped transition section at the entrance of the San Tomas Creek to the confluence basin must be based on economic considerations. Only the hydraulic factors that would influence this decision have been presented in this report.
5. The wall height of the stilling basin should be increased by at least a foot--by a foot and a half on the east side if the plane walled transition section is used.
6. The walls of the channel should be lined with riprap for a short distance downstream from the transition section below the Monroe Street bridge. About two feet of riprap should also be placed above the walls of the transition section.

DRAWINGS AND PHOTOGRAPHS



SAN TOMAS AQUINO UNIT V

PLAN VIEW OF ORIGINAL DESIGN

FIGURE 1

DRAWN: M.W.B.

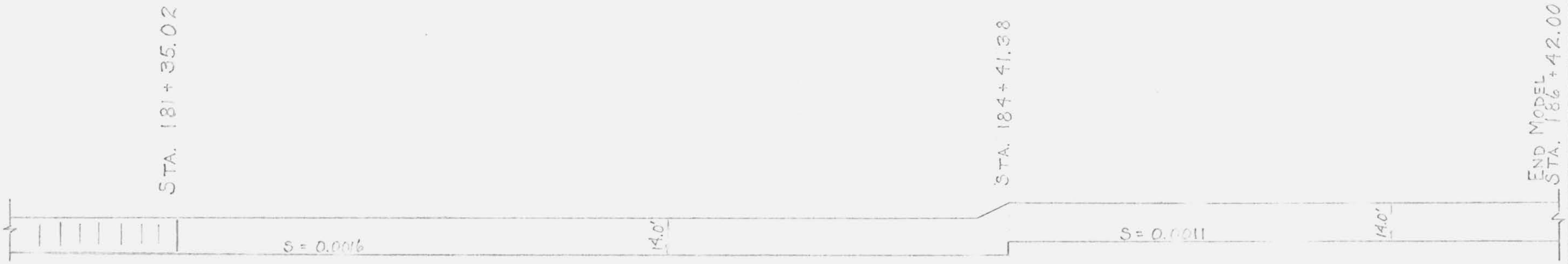
SAN TOMAS AQUINO UNIT V

ELEVATIONS OF SAN TOMAS AND SARATOGA CREEKS

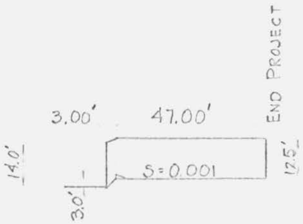
FIGURE 2

DRAWN: M.W.B.

SCALE: 1" = 60'



ELEVATION OF SAN TOMAS CREEK



ELEV. OF SARATOGA CREEK

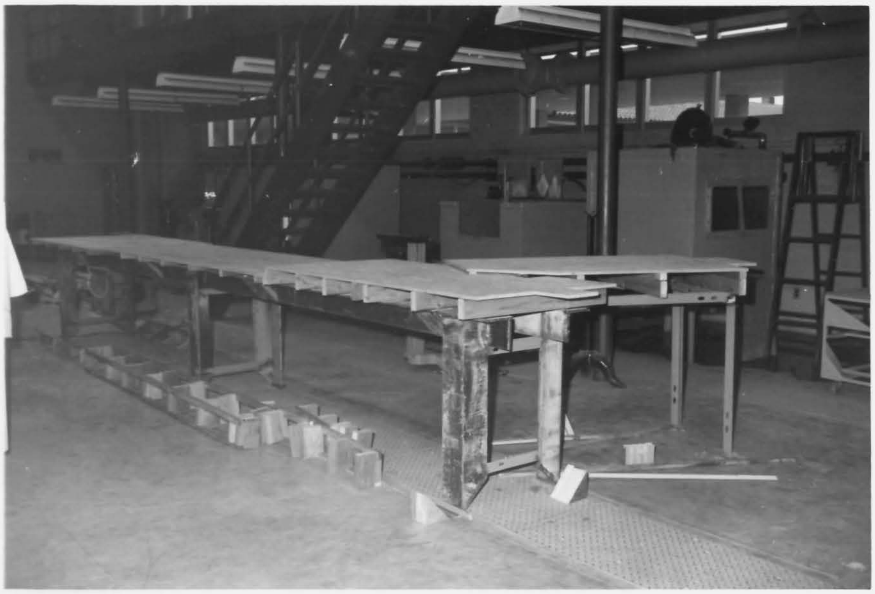


Fig. 3

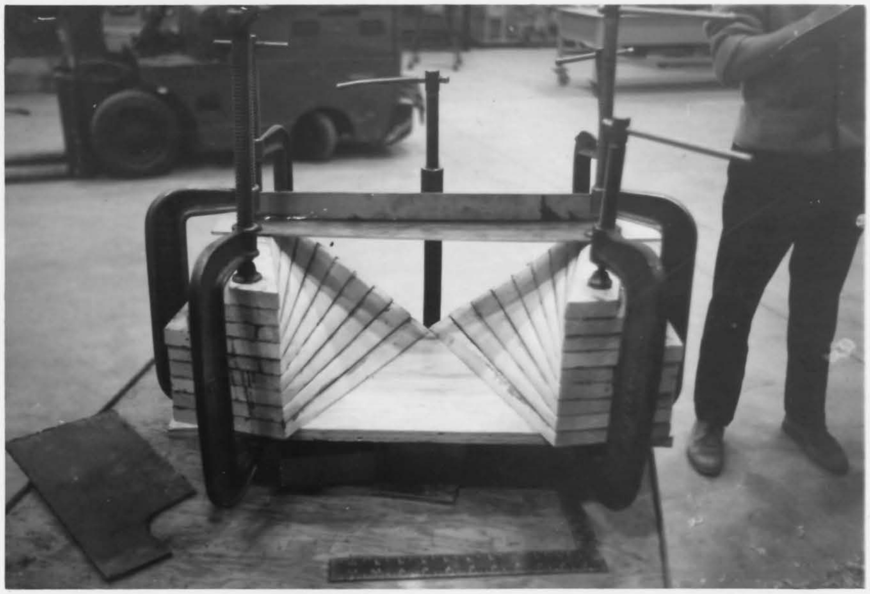


Fig. 4

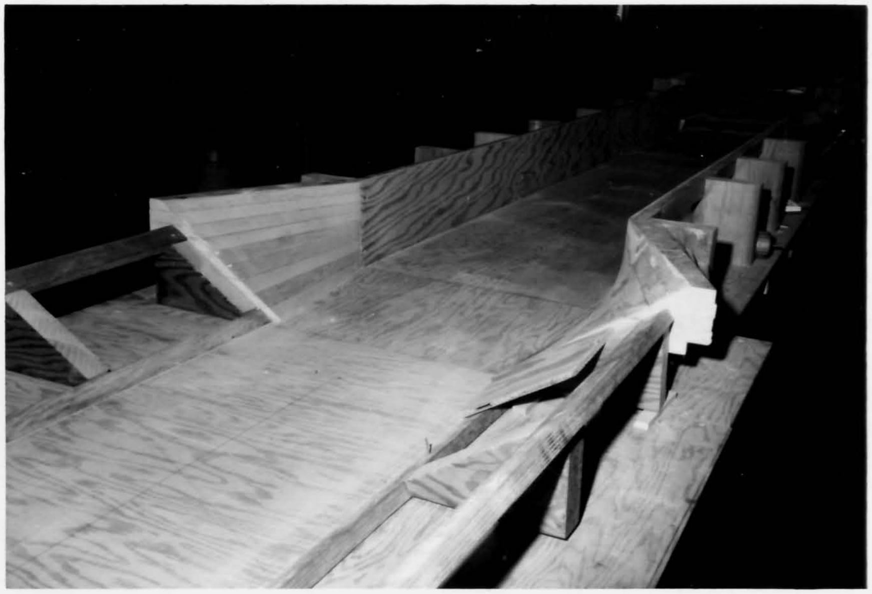


Fig. 5

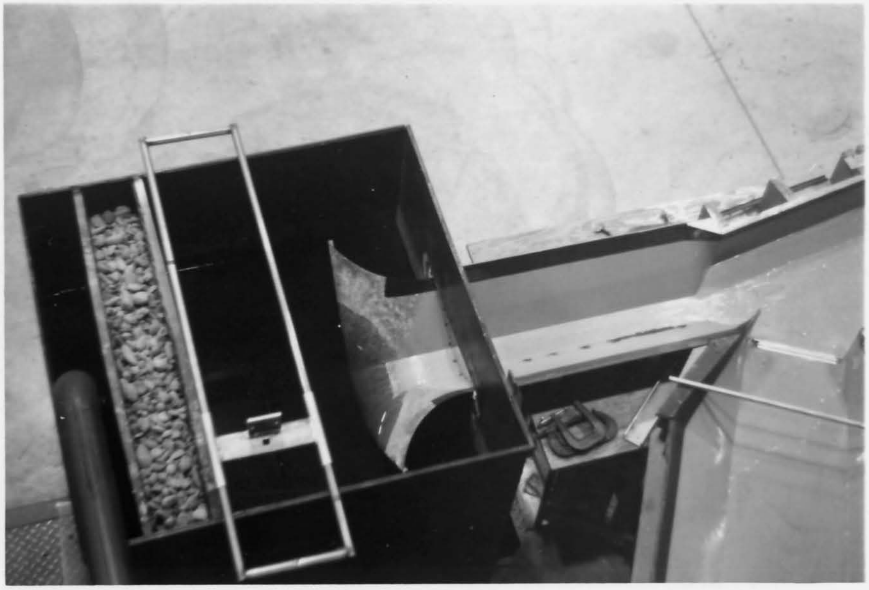


Fig. 6

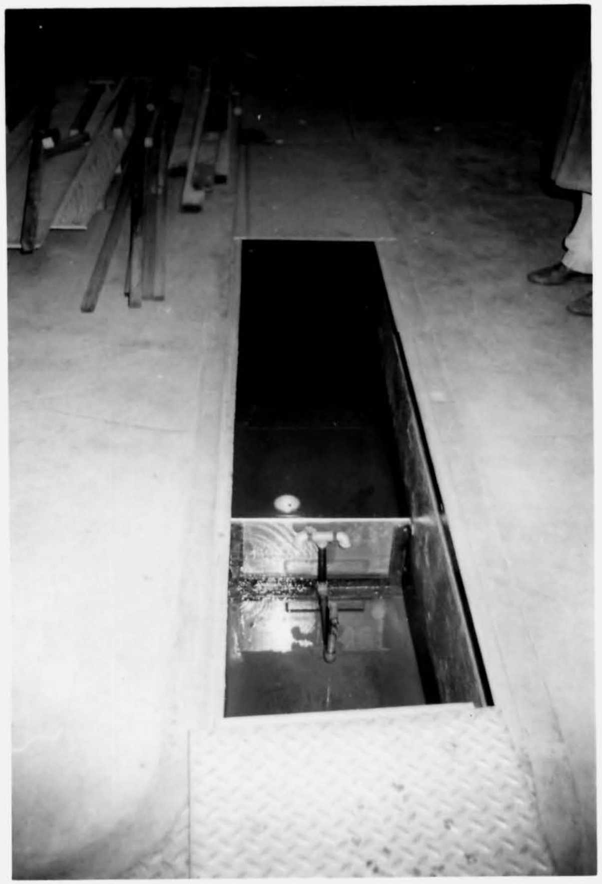


Fig. 7

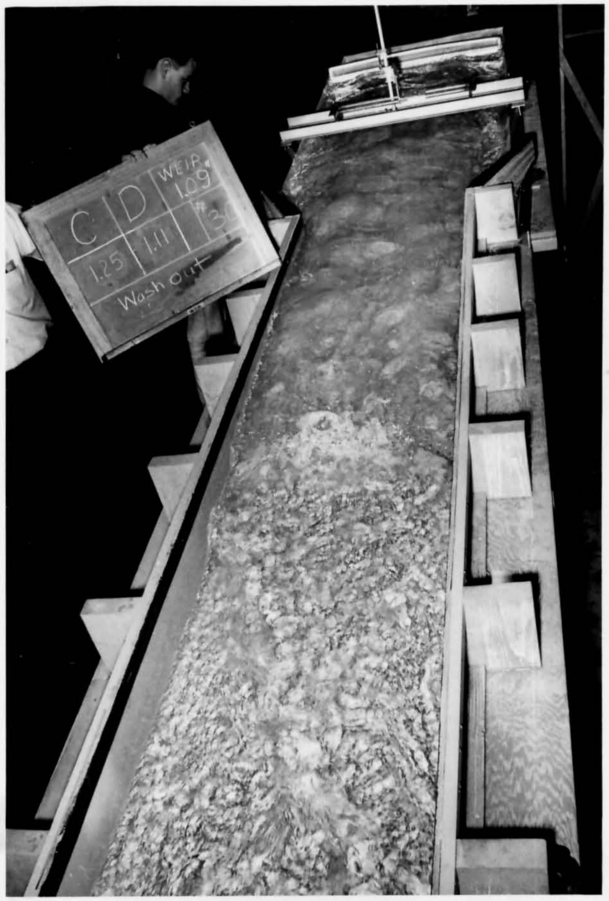


Fig. 8

Washout

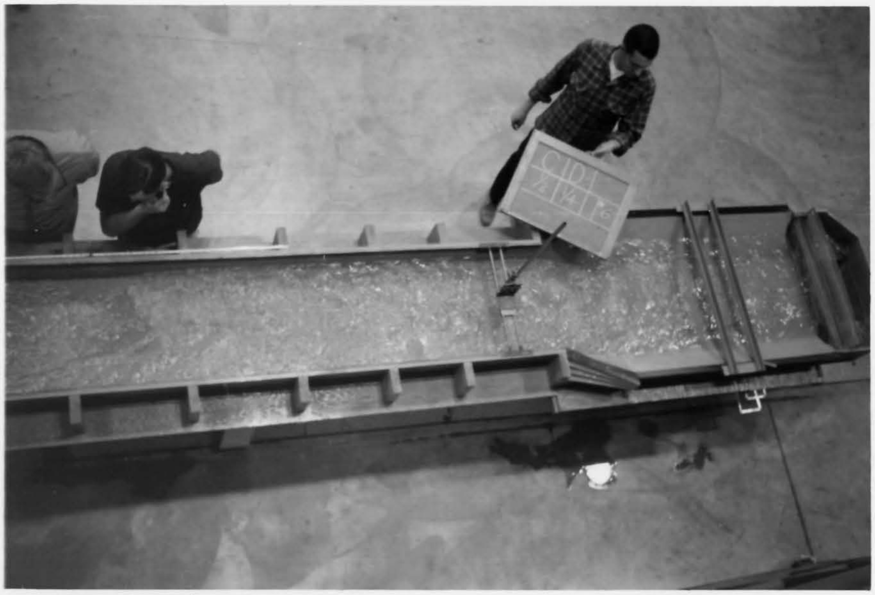


Fig. 9

2300 cfs Original Design

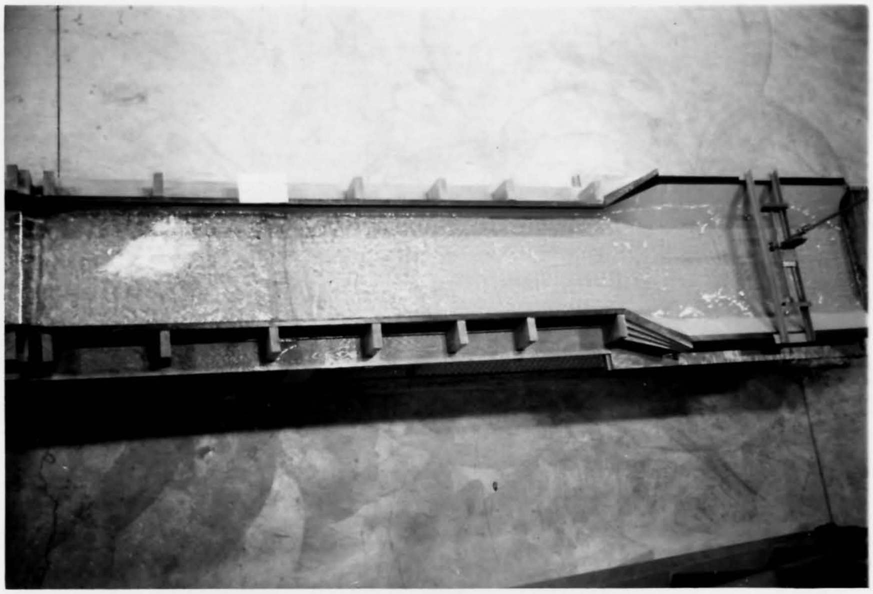


Fig. 10

2500 cfs Final Design

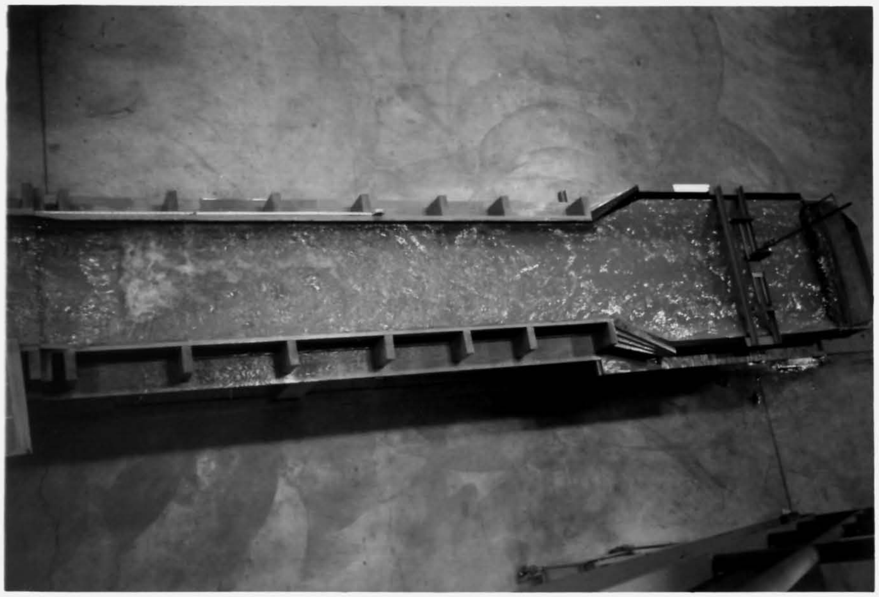


Fig. 11

7500 cfs   Original Design

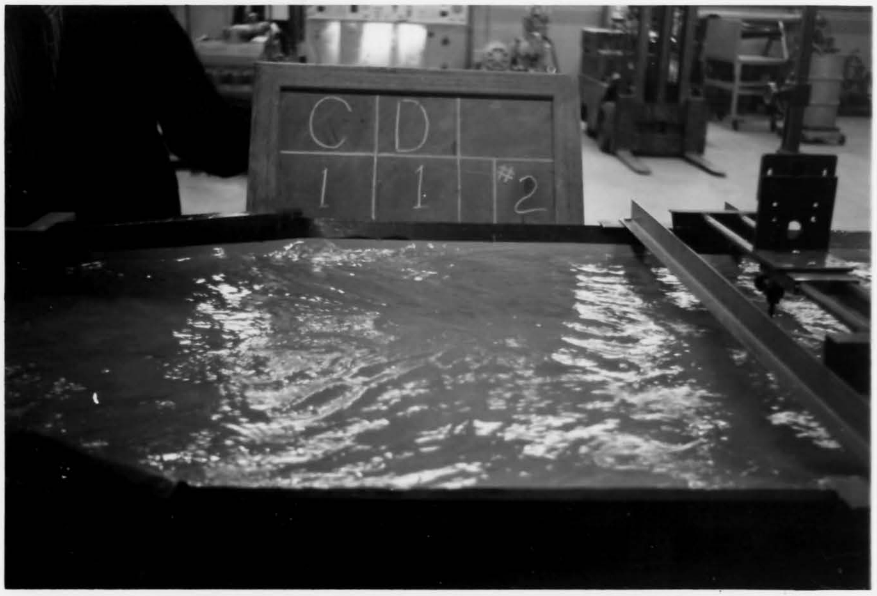


Fig. 12

6400 cfs   Original Design

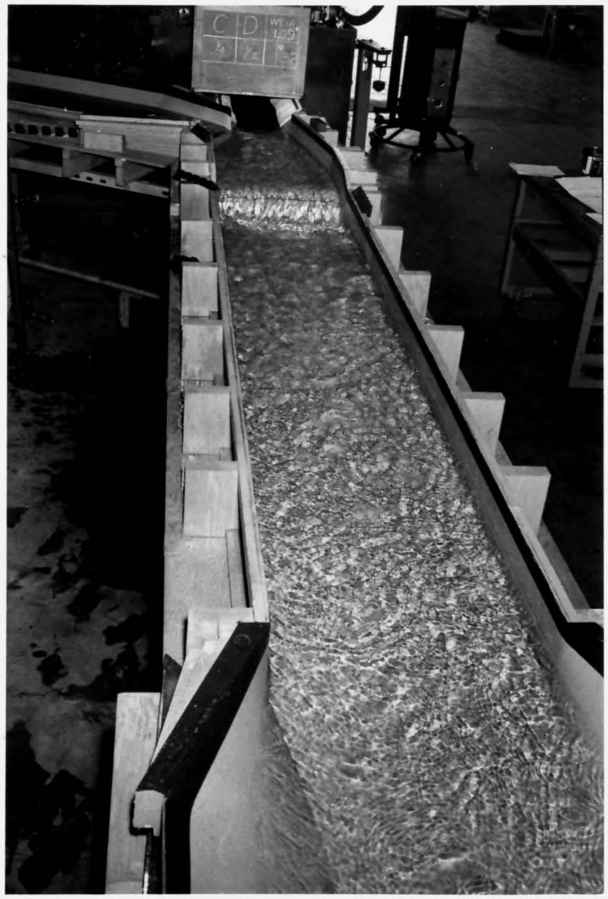


Fig. 13
2500 cfs.
Weir on
Drop Structure

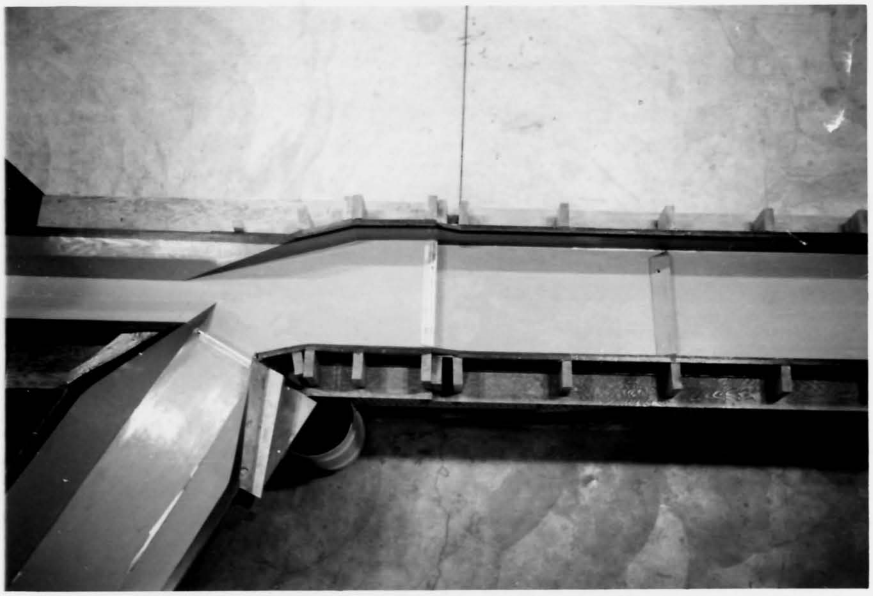
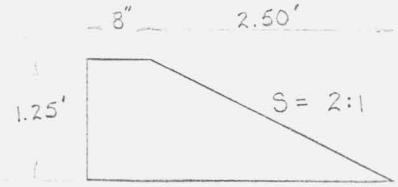


Fig. 14

Final Design

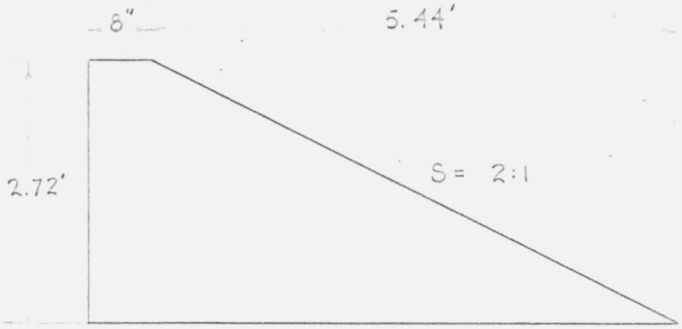
FIGURE 15



WEIR AT SARATOGA CR. OUTFALL

SCALE: 0.5" = 1'

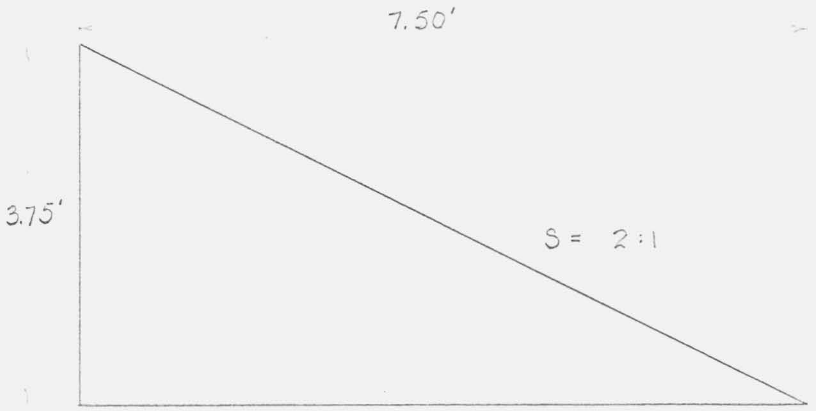
FIGURE 16



WEIR AT
SAN TOMAS CR.
DROP STRUCTURE

SCALE: 0.5" = 1'

FIGURE 17



SILL - 104' DOWNSTREAM
FROM SAN TOMAS CR.
DROP STRUCTURE

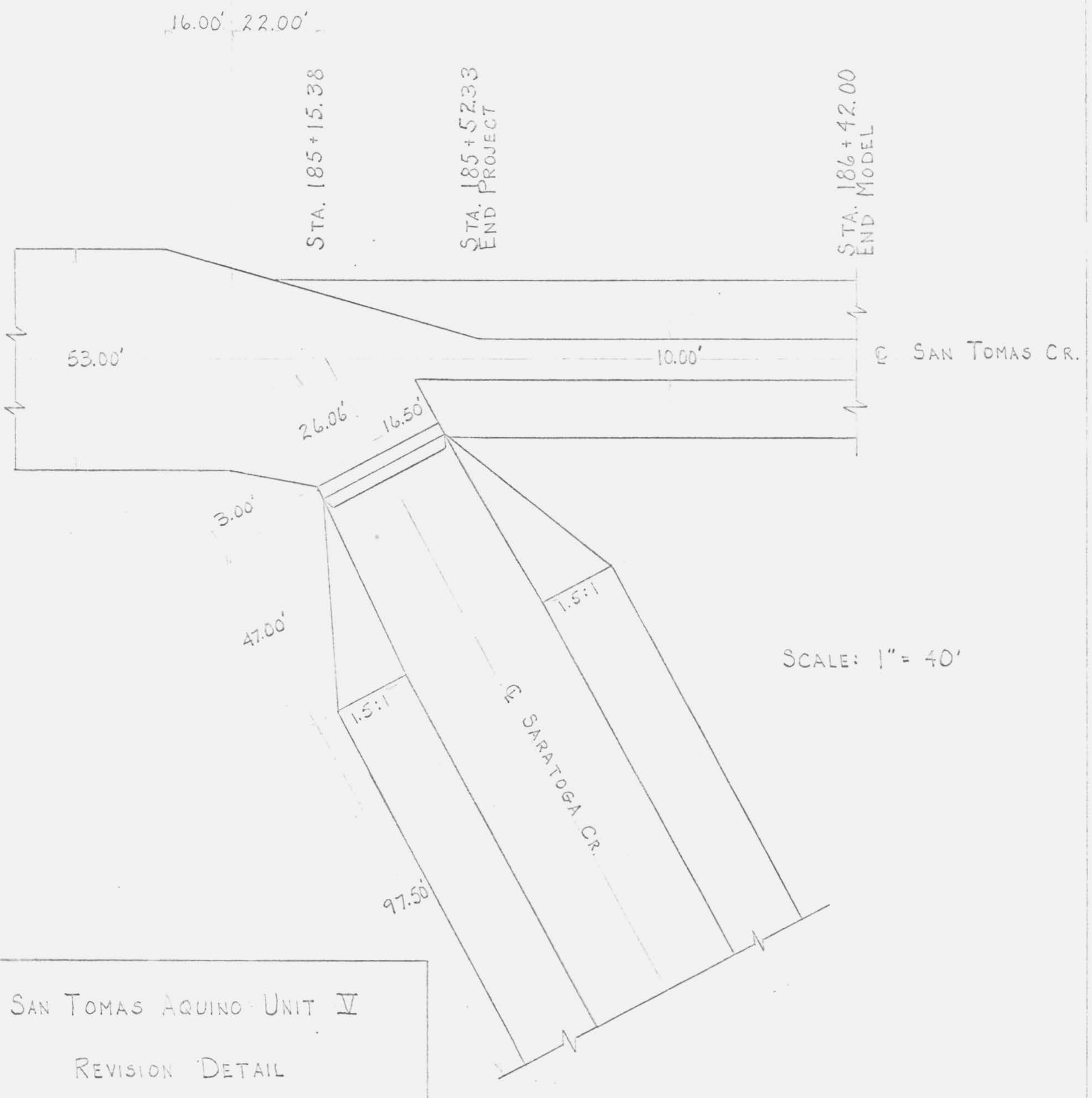
SCALE: 0.5" = 1'

SAN TOMAS AQUINO UNIT V

WEIR AND SILL DETAILS

FIGURES 15, 16, 17

DRAWN: M.W.B.



SAN TOMAS AQUINO UNIT V

REVISION DETAIL

FIGURE 18

DRAWN: M.W.B.

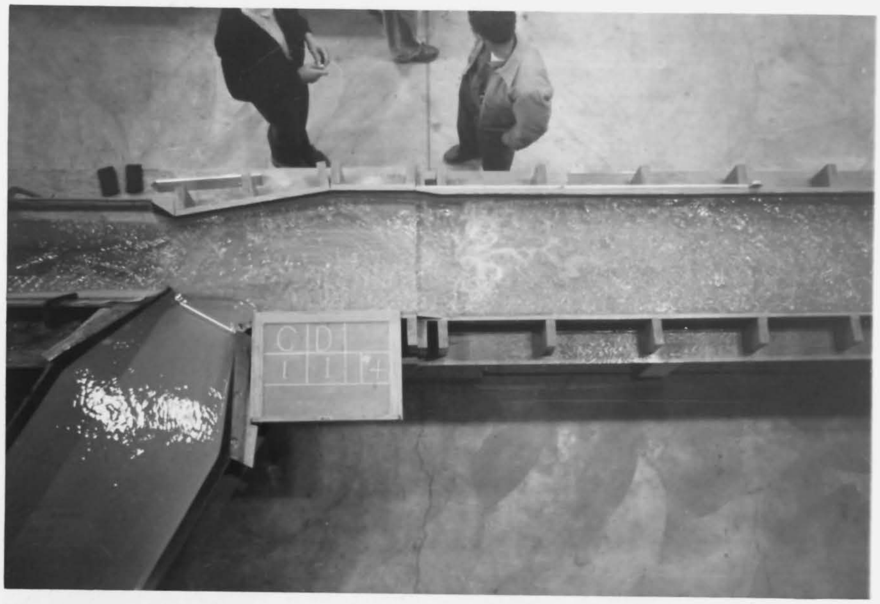


Fig. 19

6400 cfs Original Design

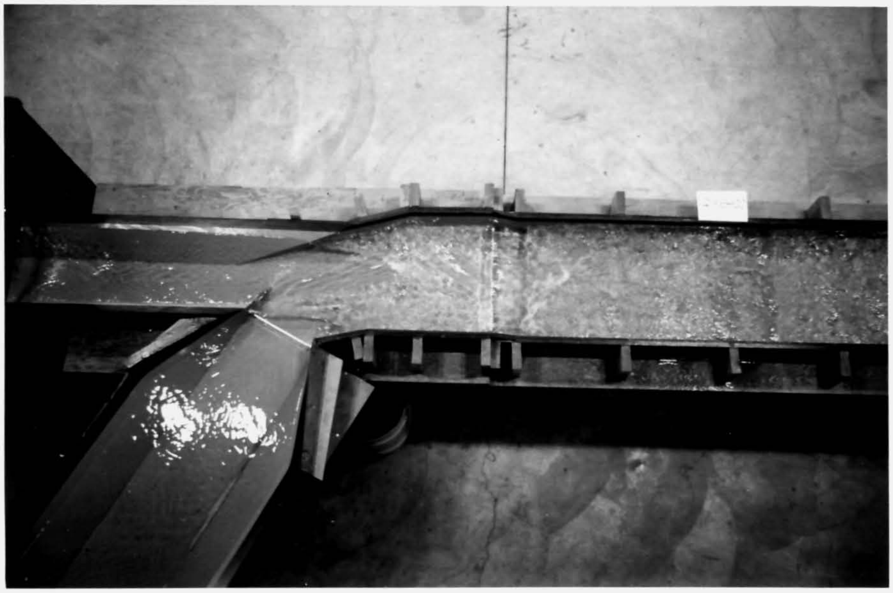


Fig. 20

6400 cfs Final Design

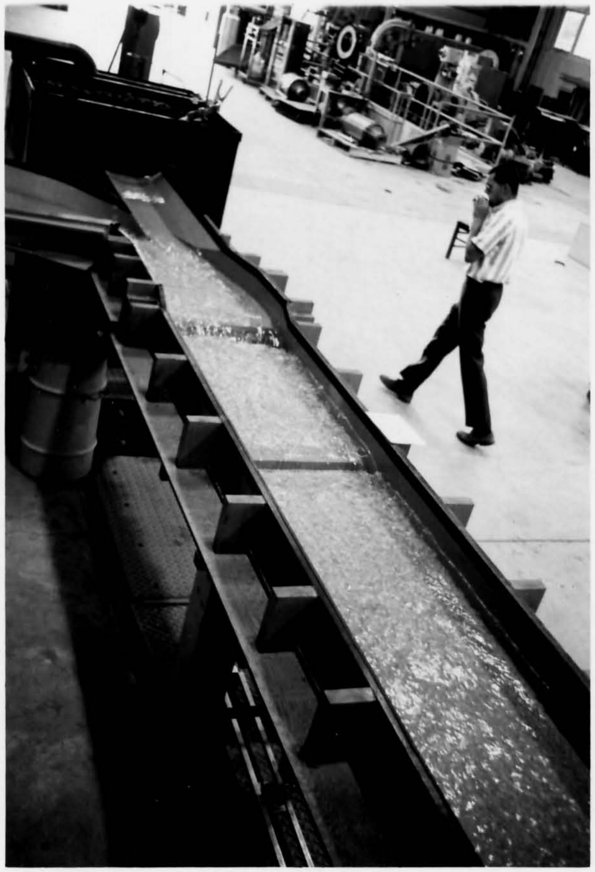


Fig. 21

2500 cfs

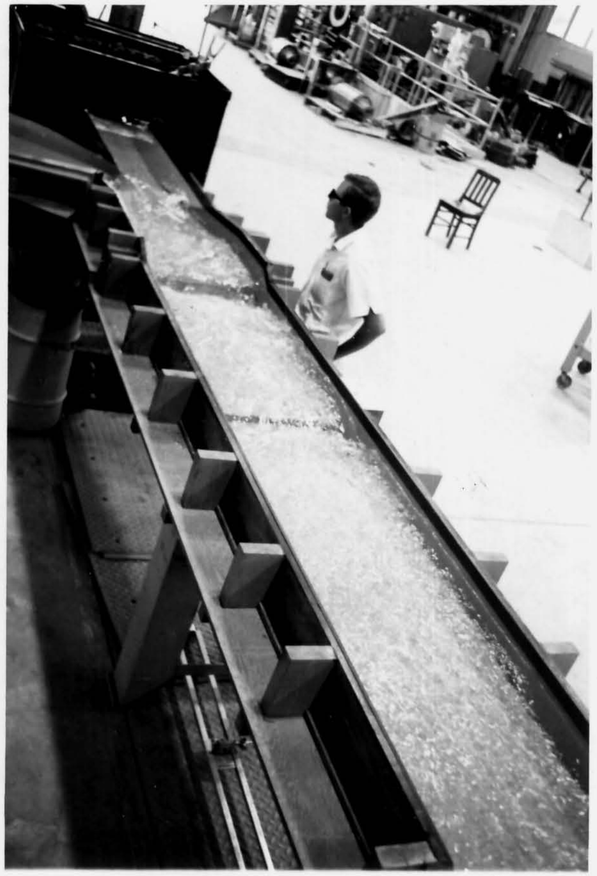


Fig. 22

6400 cfs

Final Design

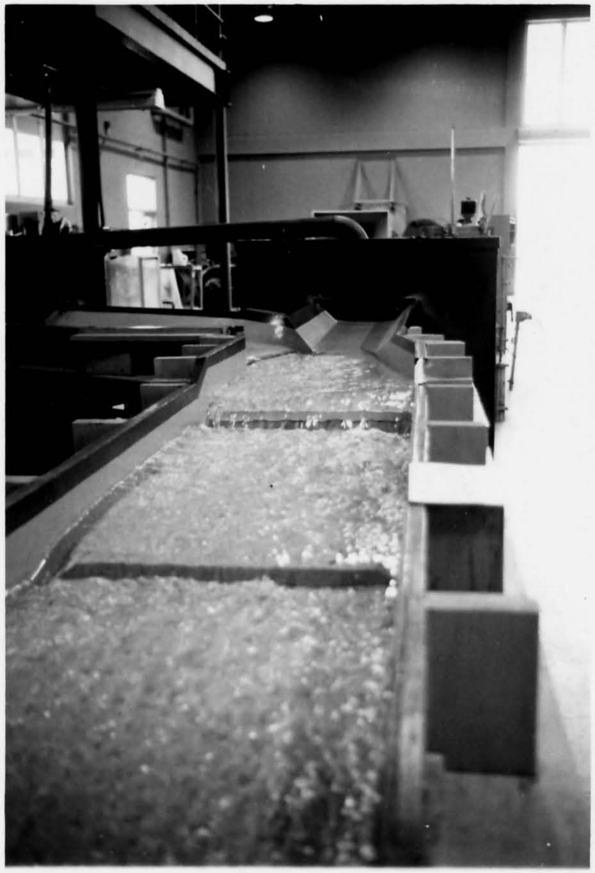


Fig. 23

2500 cfs

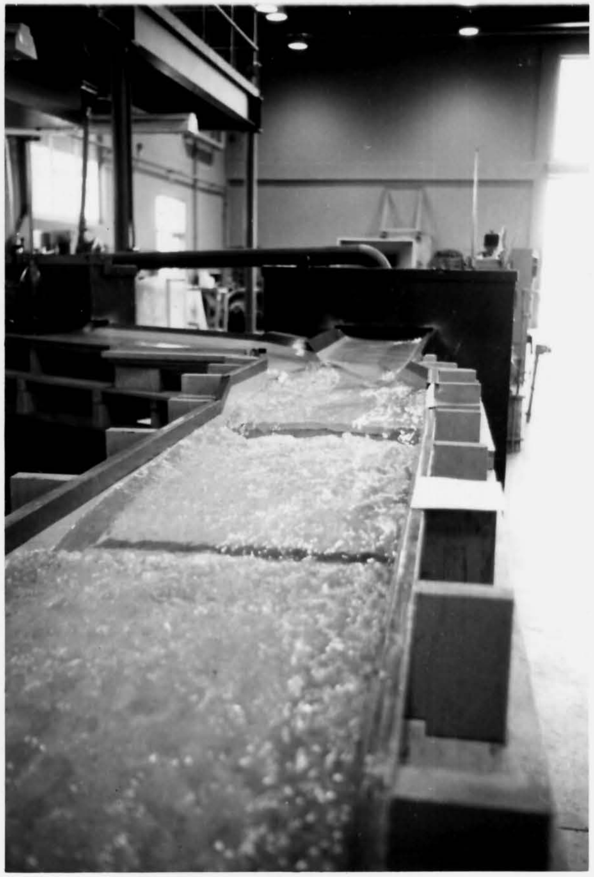


Fig. 24

6400 cfs

Final Design

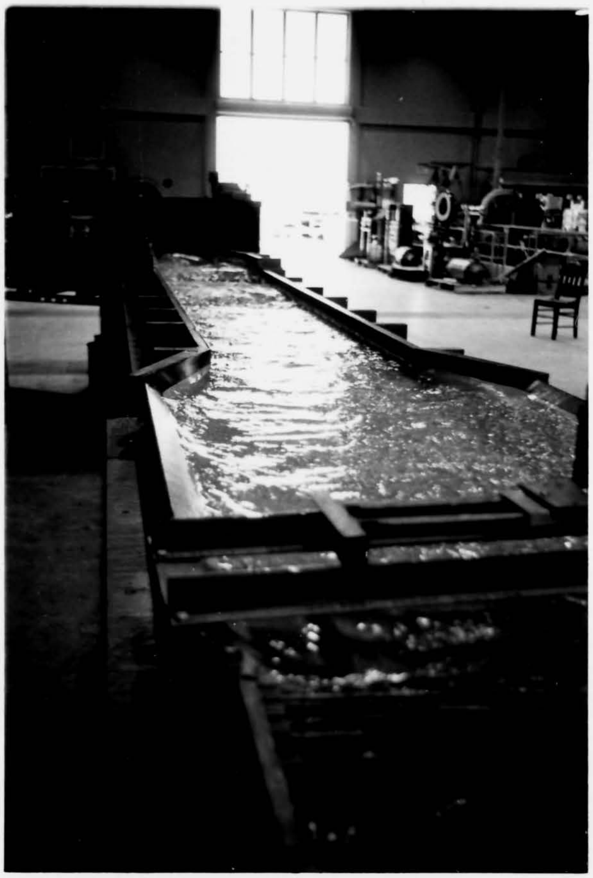
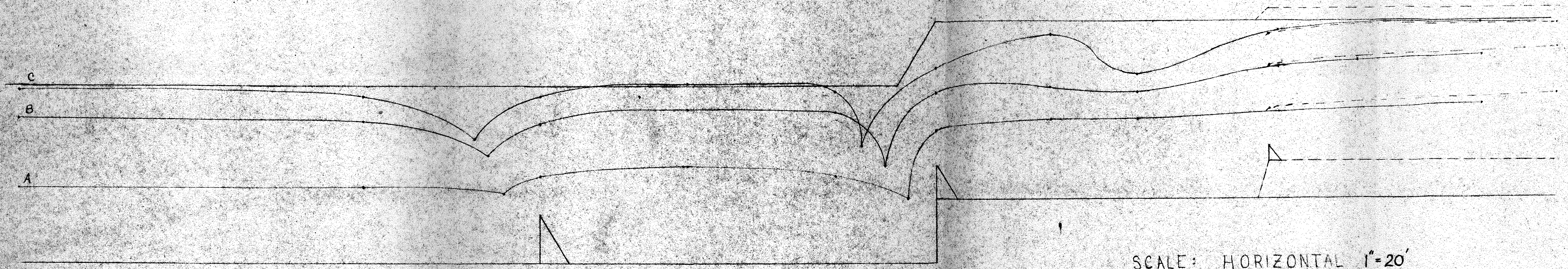


Fig. 25

6400 cfs   Final Design

SARATOGA - SAN TOMAS MODEL STUDY
SURFACE PROFILE
A - 1600 CFS B - 4800 CFS C - 7500 CFS
DASHED SECTION - SARATOGA CREEK
DR. BY | O.A. Robertson | 1 JUN 65



SCALE: HORIZONTAL 1" = 20'
VERTICAL 1" = 6'

FIG. 26